CONSTRUCTION QUALITY ASSURANCE REPORT

DISPOSAL MODULE 3.1 LINER SYSTEM

NORCAL WASTE SYSTEMS
HAY ROAD LANDFILL

REVISION 1

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1.0 INTRODUCTION

1.1 Overview

Norcal Waste Systems Hay Road Landfill, Inc. (NWSHRLI), a subsidiary of Norcal Waste Systems, Inc. (Norcal), owns and operates the Norcal Waste Systems Hay Road Landfill (NWSHRL). Disposal Module (DM) 3.1 is a Class II waste management unit that was constructed in accordance with the project technical specifications, construction drawings, and construction quality assurance (CQA) plan. This construction was also completed in accordance with Waste Discharge Requirements (WDRs) Order No. R5-2003-0118, and the applicable requirements of federal Subtitle D regulations and Title 27 of the California Code of Regulations (CCR). The project site location is shown on Figure 1.

This CQA Report documents construction activities and CQA monitoring and testing for construction of the DM-3.1 base liner system., Golder Associates Inc. (Golder) provided the CQA services for the construction of this base liner system. Following our original submittal in August 2008, Golder has revised this CQA Report to address comments made by the Central Valley Regional Water Quality Control Board (CVRWQCB).

1.2 Project Description

DM-3.1 measures approximately 8-acres in area and is located immediately south of DM-4. DM-3.1 drains to a single sump at the eastern end of the landfill. Figure 2 shows the site plan and relative location of DM-3.1. Figure 3 shows the grading plan for DM-3.1 liner system.

The DM-3.1 base grades slope at 2 percent toward leachate collection lines oriented in an east-west direction. These leachate collection lines slope at 1 percent toward the eastern perimeter berm. The eastern perimeter berm is inclined at 2H:1V (horizontal to vertical) along the interior slope and inclined at 3H:1V along the exterior slope. The northern end of the DM-3.1 primary liner system ties into the existing DM-4.1 and DM-4.2 base liner systems.

The base liner system is a double-composite liner system as described in the Liner Performance Demonstration Report prepared by Golder (April 15, 2003). The base liner containment system is comprised of the following components (from the bottom up):

- General Earthfill (the upper 6-inches comprise of fine-grained soils);
- Secondary 60-mil HPDE geomembrane (double-sided textured);
- Leak detection geocomposite;
- 2.5 feet of primary compacted clay liner ($k \leq 1 \times 10^{-7}$ cm/s, excluding the lower 6-inches);
- Primary 60-mil HPDE geomembrane (single or double-sided textured);
- 6-inch thick leachate collection and removal system (LCRS) gravel;
- 8-oz. geotextile filter layer; and
- 1-foot thick operations soil layer.

The side-slope liner system is a single-composite liner system. The side-slope liner containment system is comprised of the following components (from the bottom up):

- General Earthfill;
- Capillary Barrier 40-mil HDPE Geomembrane;

- Geosynthetic Clay Liner (GCL);
- Primary 60-mil HPDE geomembrane (single or double-sided textured);
- LCRS geocomposite; and
- 1.5-feet of operations soil.

1.3 Contractors

Construction of the DM-3.1 base liner system was performed by Boston Pacific Inc. (BPI) of Dixon, who acted as the general contractor. The installation subcontractor for the geosynthetic liner system was D&E Construction (D&E) of Visalia, California. The 60-mil HDPE primary geomembrane, 60-mil HDPE secondary geomembrane, and 40-mil HDPE capillary barrier geomembrane were provided by Poly-Flex, Inc., Grand Prairie, Texas. The geocomposite and geotextile were provided by Skapps Industries out of Athens, Georgia. GCL was provided by CETCO Lining Technologies out of Lovell, Wyoming. The leachate extraction system was installed by Advance Wind, Solar, Hydro Power, Inc. (Advanced Power), Redwood Valley, California. The gas collection system was installed by LFG Control, Inc. (LFGC), from Antioch, California. Surveying for the project was completed by Bellecci & Associates, Inc. under subcontract to BPI.

1.4 Construction Quality Assurance

Golder provided CQA monitoring and testing services for the DM-3.1 base liner construction project according to the CQA Plan approved by the CVRWQCB. The CQA services consisted of observing, testing, and documenting the construction activities to verify compliance with the construction drawings and specifications. The CQA services included, but were not limited to:

- Review of manufacturer's submittals and conformance testing of the geosynthetic products;
- Testing of the construction materials used for the general earthfill, low-permeability soil liner, LCRS gravel, and operations soil; and
- Observation of the geosynthetics installation and testing of the field seams for the HDPE geomembrane.

Golder subcontracted with Peak Engineering to provide the lead CQA technician. All CQA activities were completed under Golder's supervision. Peak Engineering provided an on-site CQA technician from May 12, 2008 until August 7, 2008. Mr. William Hensley and alternately, Mr. John Hensley, provided the lead CQA observation and testing in the field. Ms. Heather Kuoppamaki provided staff support throughout the project. Mr. Peter Bowers, P.E., provided project supervision as the CQA Engineer-of-Record.

Photographs documenting key components and activities of the construction process were taken on a regular basis. Selected photographs are included in Appendix A.

Daily field monitoring reports were prepared throughout the construction to document the construction and the CQA observation and testing. The field monitoring reports are included in Appendix B.

1.5 Project Documents

All work for the DM-3.1 base liner system was performed according to the construction drawings and specifications, which are listed below:

- "Construction Drawings, Norcal Waste Systems Hay Road Landfill, Disposal Module 3.1, Base Liner System, Solano County, California," prepared by Golder Associates, dated March 2008.
- "Construction Specifications, Disposal Module 3.1 Base Liner System, Norcal Waste Systems Hay Road Landfill, Vacaville, California," prepared by Golder Associates, dated March 2008.
- "Construction Quality Assurance Plan, Disposal Module 3.1 Base Liner System, Norcal Waste Systems Hay Road Landfill, Vacaville, California," prepared by Golder Associates, dated March 2008.

1.6 Design Changes and Clarifications

Generally during the course a project, design changes and/or clarifications are processed to facilitate the construction process. However, no design changes or clarifications were needed during the construction of the DM-3.1 base liner system.

1.7 Surveying and Preparation of Record Drawings

Bellecci & Associates, Inc., under the supervision of Charles N. Capp, registered land surveyor, performed surveying for the project. Bellecci & Associates, Inc. established control points in the field for use by the contractor. Based on the control points, BPI performed construction grade control using Global Positioning System (GPS) technology. Bellecci & Associates, Inc. completed as-built surveys using the 50-foot grid system presented in the Drawings to determine the as-built elevations of each layer. As-built surveys were completed for the following:

- Top of general earthfill;
- Top of low-permeability soil liner;
- Lysimeter, leak detection, and LCRS piping;
- Top of LCRS gravel (recorded by BPI); and
- Top of operations soil.

The record drawings prepared by Bellecci & Associates, Inc. are presented in Appendix C.

The location of each HDPE geomembrane panel was determined in the field using a measuring wheel. The record drawings for the HDPE geomembrane panels (primary and secondary layers) were prepared by D&E and reviewed by Golder. These drawings are also presented in Appendix C.

2.0 SUBGRADE PREPARATION AND GENERAL EARTHFILL

The subgrade preparation required for the construction of the DM-3.1 base liner system consisted placing compacted general earthfill to the lines, grades and tolerances specified in the construction drawings. Approximately 59,100 cubic yards (cy) of general earthfill was excavated from the site borrow area and then used as general fill for base and eastern perimeter berm associated with DM-3.1. Additionally, the perimeter berm was extended to the southeast corner of the landfill and approximately 1,000 feet along the southern landfill perimeter. This berm extension consisted of approximately 52,100 cy of general fill. The borrow soils predominately consist of clays and silty clays.

General earthfill placement began on May 14, 2008. The general earthfill was excavated using a Kamatsu 400 excavator and hauled to DM-3.1 using tandem belly-dump trucks. Excavation occurred from the soil borrow area located west of the landfill. Standing water in the borrow area was pumped out beginning in mid-April and dewatering trenches were excavated in the borrow area. In general, the soils were excavated at moisture contents that met specification requirements. However, some moisture conditioning took place when the soils were too wet or dry to meet compaction requirements. Compaction was performed with a Caterpillar 815 sheepsfoot compactor. General earthfill placement for the cell subgrade was completed on June 9, 2008. The general earthfill for the berm extension continued until July 10, 2008.

CQA procedures for the general earthfill consisted of laboratory testing, monitoring placement methods, moisture conditioning, and determination of compaction using nuclear moisture-density testing methods (ASTM D6938). Laboratory testing of the general earthfill material consisted of Proctor compaction tests (ASTM D1557). Appendix D.1 includes the proctor compaction curves for the general earthfill. A summary of the in-situ density testing is presented in Appendix D.2. 233 compaction tests were performed on an estimated 111,200 cy of soil, resulting in a testing frequency of 47 cy/test. The results of the compaction tests each measured a relative compaction of at least 90 percent. The project specifications required a minimum relative compaction of 90 percent in accordance with ASTM D 1557. The compaction test results indicated that the general earthfill was placed and compacted in accordance with the project specifications.

The general earthfill material was compacted to provide a firm and unyielding surface to support the liner system. At the completion of the placement and compaction, the exposed soils at the surface were systematically examined by Golder's CQA Technician to verify that the upper surface of the liner subgrade consisted of clay, silty clay, and/or sandy clay classified as CH, CL, or SC in accordance with the Unified Soil Classification System.

A topographic survey was prepared by Bellecci & Associates, Inc. Point data was also submitted and verified for compliance with the design grading tolerances by Golder. The survey elevations are indicated on the as-built topographic drawing, presented in Appendix C.

3.0 LOW-PERMEABILITY SOIL LINER

3.1 General

The low-permeability soil material was obtained from on-site clay soils contained in the borrow area located west of the landfill. Golder completed a borrow investigation on February 7, 2008. Laboratory tests were performed on selected samples to verify suitability of the on-site soil for use as a low-permeability soil liner material. In addition, a test pad and field infiltration test was completed at the beginning of the project to verify that the proposed equipment and handling procedures would result in low-permeability soil liner that met the compaction and permeability requirements.

3.2 Test Pad Construction and Field Infiltration Measurement

3.2.1 Test Pad Construction

A test pad was constructed on May 14, 2008 to establish placement and compaction procedures for the new low-permeability soil liner material used for the primary liner. The test pad was constructed in an area measuring approximately 30 feet by 70 feet located southwest of DM-3.1 (see Figure 3). Following completion of the test pad and the field permeability testing, the test pad material was incorporated into the southern perimeter berm extension as general fill.

The test pad consisted of four 6-inch thick compacted lifts. The borrow soils were moisture conditioned in the borrow area using a water truck and a tractor disc. The soils were excavated using a Kamatsu 400 excavator and hauled to the test pad using belly-dump trucks. The soils were placed and compacted in 6-inch thick lifts using a Caterpillar 815 sheepsfoot compactor.

Samples of the low-permeability soil liner were obtained on each lift and laboratory testing completed to measure moisture content (ASTM D2216), particle-size distribution (ASTM D1140), Atterberg Limits (ASTM D4318), modified Proctor density (ASTM D1557), and hydraulic conductivity (ASTM D5084). The results of this testing are summarized in Appendix E.1.

Compaction testing on the test pad was completed using a nuclear density gauge (ASTM D6938). The results of this testing are included in Appendix E.2.

3.2.2 Evaluation of In-Situ Permeability

In-situ permeability was evaluated by obtaining four pairs of relatively undisturbed Shelby tube samples of the completed test pad soils for laboratory permeability testing. In addition, a two-stage borehole infiltration test, referred to as the Boutwell Permeameter (ASTM D6391), was performed following completion of the test pad. The Boutwell Permeameter test consisted of six borehole permeameters measuring 4-inches in diameter. The permeameters were installed in two rows of three, separated to allow a radius of influence of approximately 6 to 7 feet (a spacing of 12 to 14 feet apart). A sealed, control permeameter was installed in the center of the test permeameter layout. This permeameter was sealed and had no radius of influence. The purpose of the control permeameter was to measure environmental fluctuations, such as barometric pressure and temperature, to allow for corrections to the other permeameter readings.

Completion of the field infiltration test using the Boutwell Permeameter can involve one or two stages. The first stage involves placement of the permeameter flush with the bottom of the test hole such that infiltration occurs through the bottom of the borehole. At the end of this first stage, an upper-bound vertical permeability value is calculated by conservatively assuming that infiltration occurs vertically through the bottom of the test hole. This permeability is a conservative upper-bound value because the infiltration rate actually involves some radial flow from the bottom of the permeameter. Therefore, the true vertical permeability is less than this upper-bound value. In many cases, the field infiltration is halted at the end of the first stage if the upper bound permeability value is less than the permeability specification requirement.

For this project, Stage 2 tests were also completed, even though the result of the Stage 1 tests indicated passing permeability values. The second stage of the test involved advancing the test hole below the base of the permeameter and then resuming measurement of the infiltration rate. The second stage primarily measures horizontal permeability, although there is a vertical and radial component of flow. By assuming that flow is only horizontal, the second stage measures an upper-bound horizontal permeability. After calculating the upper bound vertical and horizontal permeability values, the true vertical and horizontal permeabilities can be determined using the equations developed by Boutwell (Trautwein and Boutwell, 1994) if the soils are homogeneous.

For the DM-3.1 base liner project, the results of the first stage measured an average upper-bound permeability of 3.07×10^{-8} cm/s, which is below the specification requirement of 1×10^{-7} cm/s. The results of the second stage indicated an upper bound horizontal permeability of 1×10^{-7} cm/s or less. A true average vertical permeability of 5.8×10^{-9} cm/s was calculated. The Boutwell Permeameter test results are summarized in Appendix E.3.

The four laboratory permeability tests for the test pad soil samples measured permeability values ranging from 3.8×10^{-9} to 1.5×10^{-8} cm/s with an average permeability of 7.9×10^{-9} cm/s. As a result of these laboratory measured values, the compaction window was not modified from the clay borrow investigation as discussed below.

The results of the field and permeability tests indicate that the borrow materials are capable of being placed and compacted to achieve a permeability of less than 1×10^{-7} cm/s. In addition, the laboratory permeability tests correlate very closely to the Boutwell permeameter tests. The average permeability was measured to be 6.6×10^{-9} cm/s in the laboratory, and the field permeability results measured an average vertical permeability of 5.8×10^{-9} cm/s, a difference of 8×10^{-10} cm/s.

The plasticity laboratory test between the test pad and the borrow investigation correlated closely. The average liquid limits were 40 and 37 and the average plastic indices were 23 and 22, respectively for the test pad and clay borrow investigation.

For the past eleven years at the Hay Road Landfill, the compaction window has been defined by a minimum degree of saturation of 83 percent, a minimum relative compaction of 90 percent, and a moisture content of 16 to 22 percent. Historically, the measured maximum dry densities have been around 118 pcf, resulting in a minimum dry density requirement of 106 pcf. For the DM-3.1 test pad, we measured a maximum dry density of 118.6 pcf, which correlates to the maximum dry density from the clay borrow investigation of 117.9 pcf. The compaction window for the DM-3.1 test pad and low-permeability soil liner was based on 117.9 pcf, resulting in a minimum dry density of 106.1 pcf based on a 90 percent relative compaction. The compacted density achieved for the test pad exceeded 107.5 pcf.

3.3 Low-Permeability Soil Liner Construction

The low-permeability soil liner is a 2.5-foot thick layer of compacted low-permeability soil in which the upper two feet is required to have a permeability of 1×10^{-7} cm/s or less. The soils used to construct this layer were obtained from the borrow area located west of the landfill. The low-permeability soil liner was constructed from June 23, 2008 through July 10, 2008.

The low-permeability soil consisted of brown, silty clay classified as a CH or CL in accordance with the Uniform Soil Classification Systems (USCS) per ASTM D2487. The soils exhibited an average Liquid Limit (LL) of 39, an average Plastic Index (PI) of 24 and average fines content (minus No. 200 sieve) of 79 percent.

The low-permeability soil liner was constructed directly on top of the leak detection geocomposite. The first lift measured 12-inches in thickness to prevent construction damage to the underlying geosynthetic layers. The lower 6-inches of the first lift was placed as a foundation layer for the overlying 2-feet of low-permeability soil liner. Although the foundation layer was not tested for compaction or permeability, it was tested for Atterberg Limits and grain-size properties to establish that the material is the same as that used for the soil liner. The quantity of foundation layer material was estimated at 6,140 cy and the low-permeability soil liner was estimated at 24,560 cy, for a combined total of 30,700 cy.

The low-permeability soil liner material was excavated from the borrow area at moisture contents within the specified compaction window. Moisture was maintained in the placement area using a water truck. The soil was excavated with a Komatsu 400 and hauled to DM-3.1 using tandem belly dump trucks. Following the initial 12-in lift, the soils were placed in 6 to 8-inch thick loose lifts and compacted with a Caterpillar 815 sheepsfoot compactor. Final grading was completed using a Caterpillar 140G grader.

The compaction window consisted of the same window used for the low-permeability soil liner test pad. This compaction window was defined by a minimum moisture content of 17 percent, a minimum relative compaction of 90 percent, and a minimum degree of saturation of 83 percent.

CQA procedures consisted of monitoring placement, moisture conditioning, and measurement of in-situ moisture-density using a nuclear density gauge (ASTM D6938) and the drive cylinder method (ASTM D2937). In addition, samples of the low-permeability soils were obtained for laboratory testing including moisture content (ASTM D2216), particle-size distribution (ASTM D1140), Atterberg Limits (ASTM D4318), modified Proctor density (ASTM D1557), and hydraulic conductivity (ASTM D5084). The results of this testing are summarized in Appendix F.1. A summary of the in-situ moisture density testing is presented in Appendix F.2. CQA testing frequencies are summarized in Table 1.

TABLE 1
LOW-PERMEABILITY CQA TESTING FREQUENCIES

Parameter	Test Method	Minimum Specified Frequency	Number of Tests	Actual Construction Frequency
Moisture-Density	D1557	1 Per 5,000 or change in material	8	1 Per 3,838 CY
Nuclear Moisture-Density	D6938	1 Per 250 CY	104	1 Per 236 CY
Moisture Content	D2216/D4643	1 Per 1,500 CY	21	1 Per 1,462 CY
Sand Cone, or Drive Cylinder	D1556, D2937	1 Per 20 Nuclear Density Tests	5	1 Per 20.8 tests
Particle Size	D422/D1140	1 Per 1,500 CY	21	1 Per 1,462 CY
Atterberg Limits	D4318	1 Per 1,500 CY	21	1 Per 1,462 CY
Soil Classification	D2487/2488	1 Per 1,500 CY	21	1 Per 1,462 CY
Laboratory Hydraulic Conductivity on Field Collected Sample	D5084 at 15 psi	1 Per 1,500 CY	17	1 Per 1,445 CY

On average, the soils were compacted to a dry density of 109.9 pcf and a moisture content of 19.1%.

Permeability samples were obtained in 3-inch diameter Shelby tubes and transported to Sierra Testing Laboratories in El Dorado Hills, California. The results of the permeability testing indicated measured permeabilities ranged from 1.3×10^{-9} cm/s to 6.5×10^{-8} cm/s with an average of 1.7×10^{-8} cm/s.

The top of the primary low-permeability soil liner was surveyed to verify that the design thickness and grades were achieved. The as-built plan is included in Appendix C.

The results of the CQA observations, field and laboratory testing, and surveying indicate that the primary low-permeability soil liner material was placed in general compliance with the project specifications.

4.0 LCRS, LEAK DETECTION, AND LYSIMETER GRAVELS

The leachate collection and recovery system (LCRS) consists of a 0.5-foot thick layer of 3/8-inch pea gravel spread across the floor of the cell over the 60-mil HDPE primary geomembrane. The LCRS gravel materials were also placed in the lysimeter and leak detection sumps. Additionally, LCRS collection pipes were installed on the floor. BPI began welding LCRS collection pipe on July 15, 2008 and began placing LCRS gravel on July 17, 2008. HDPE piping materials were obtained from ISCO Industries.

The LCRS gravel was supplied by Cemex located in Madison, California. Gravel was hauled to the site in transfer dump trucks and placed at the most accessible edge of the gravel placement area. As the gravel was graded, geotextile placed, and water sprayed over the gravel, the LCRS layer was covered with operations layer materials. BPI spread and graded the LCRS gravel using a Caterpillar D6 low-ground pressure (LGP) dozer operating on a base of approximately 6-inches of gravel. The dozer used global positioning system (GPS) guided survey equipment to assist with grade control.

Samples were obtained from the gravel that was delivered to the site. The samples were tested for grain-size (ASTM D422), fractured faces (ASTM D5821), and permeability (ASTM D2434). The measured permeability exceeded the minimum requirement of 1.0 cm/s and averaged 3.4 cm/s. The gravel met the maximum particle-size requirement (100 percent less than 1/2-inch minus and 100 to 85 percent minus the 3/8-inch sieve) and the maximum fines content (a maximum of 2 percent less than the U.S. No. 200 sieve). The percent finer than the U.S. No. 4 sieve ranged from 27 to 43 percent and averaged 33 percent, which exceeded the specification requirement of 30 percent or less. The design specification requirement was based on a commonly available gravel gradation that has consistently demonstrated adequate permeability. The gradation of the gravel for this project was accepted based on the satisfactory permeability characteristics of this material (measured permeability of 3.0 to 4.3 cm/s). The percentage of particles with more than one fractured face was measured between 5.1 and 11.3 percent, which was less than the 25 percent maximum value.

The CQA testing frequencies met or exceeded the CQA plan requirements and are detailed in Table 2.

TABLE 2
LCRS GRAVEL CQA TESTING FREQUENCIES

Parameter	Test Method	Minimum Specified Frequency	Number of Tests	Actual Construction Frequency
Sieve Analysis	D422/C136	1 Per Source[1] 1 Test Per 1,500 CY	7	1 Per 886 CY
Visual Classification	D2488	Continuous Observation		Continuous Observation
Hydraulic Conductivity	D2434	1 Per Source[1] 1 Test Per 3,000 CY	4	1 Per 1,550 CY
Fractured Faces (Gravel Fraction Only)	D5821	1 Per Source[1] 1 Test Per 1,500 CY	5	1 Per 1,240 CY

Based on the survey data submitted by BPI, the thickness of the LCRS gravel was generally 0.5 feet thick, averaged 0.51 feet thick and was within design tolerances.

5.0 GEOSYNTHETICS

5.1 Review of Submittals and Material Conformance Testing

Geosynthetics utilized for the DM 3.1 base liner construction project consisted of the following components:

- 60-mil double-sided textured HDPE geomembrane liner;
- Geocomposite drainage layer;
- 8-oz. geotextile filter layer; and
- Geosynthetic Clay Liner (GCL) with a separate 40-mil HDPE geomembrane backing sheet.

Golder performed conformance testing of the HDPE geomembrane, geocomposite, geotextile, and GCL materials and reviewed the manufacturer's quality control certificates prior to use of the materials on the project. Copies of the manufacturer's quality control documentation are included in Appendix H as follows:

- Appendix H.1 – HDPE Geomembrane;
- Appendix H.2 – Geocomposite;
- Appendix H.3 – Geotextile; and
- Appendix H.4 – Geosynthetic Clay Liner.

Conformance samples were obtained from the manufacturing plant or upon delivery to the site. Golder staff selected the rolls of materials for conformance sampling. Samples were shipped to Golder's Geosynthetics Laboratory in Atlanta, Georgia for conformance testing. Copies of the conformance tests results and test summaries are presented in Appendix I as follows:

- Appendix I.1 – HDPE Geomembrane;
- Appendix I.2 – Geocomposite;
- Appendix I.3 – Geotextile; and
- Appendix I.4 – Geosynthetic Clay Liner.

Golder's technicians performed an inventory of the on-site materials to confirm that the roll numbers for each of the geosynthetic components correlated to the manufacturer's submittals and shipping manifests. Copies of the material inventories prepared by Golder are presented in Appendix J as follows:

- Appendix J.1 – HDPE Geomembrane;
- Appendix J.2 – Geocomposite;
- Appendix J.3 – Geotextile; and
- Appendix J.4 – Geosynthetic Clay Liner.

The frequency of conformance testing met or exceeded minimum frequencies specified in the CQA Plan, which are summarized below:

- HDPE Geomembrane: 8 tests for 688,000 square feet (min. required frequency of 1 test/150,000 s.f. required);

- Geocomposite: 2 tests for 353,000 square feet (min. of 1 test/250,000 required);
- Geotextile: 3 tests for 333,000 square feet (min. of 1 test/150,000 required);
- Geosynthetic Clay Liner: 1 test for 26,000 square feet (min. of 1 test/150,000 required);

Based upon the manufacturer's quality control documentation and the results of the conformance tests, all of the geosynthetic materials used for the project were accepted. All of the conformance tests for the GCL, geocomposite, and geotextile met project specifications. However, eight rolls of the Poly-Flex 60-mil HDPE geomembrane were rejected for non-conformance with the thickness specification. The corresponding roll numbers are: HT2-6-08-6201, HT2-6-08-6202, HT2-6-08-6233, HT2-6-08-6234, HT2-6-08-6235, HT2-6-08-6245, HT2-6-08-6246, and HT2-6-08-6247. These rolls were bracketed by passing conformance tests.

5.2 60-Mil HDPE Geomembrane Liners

A 60-mil double-sided textured HDPE geomembrane liner was installed in the primary and secondary liner systems and in the pan lysimeter to the limits identified on the design drawings. The HDPE geomembrane was deployed following the completion of the grading and surveying of the secondary and primary low-permeability soil liners. Prior to geomembrane deployment, the subgrade was inspected to verify that it was suitable to support the geomembrane liners. Copies of the subgrade certificates are included in Appendix K.1.

The pan lysimeter and secondary HDPE geomembrane deployment began on June 10, 2008. The secondary geomembrane liner was completed on June 20, 2008. The primary HDPE geomembrane deployment began on July 9, 2008 and was completed on July 17, 2008.

The 60-mil HDPE geomembrane was deployed using an all-terrain forklift and a spreader bar. Each roll measured 22.5 feet wide by 520 feet long. A record of the deployment logs is presented in Appendix K.2. The record drawings representing the location of the liner panels were prepared by D&E and reviewed by Golder. These as-built record panel drawings are presented in Appendix C.

Golder observed the deployment and seaming of the 60-mil HDPE geomembrane installed by D&E. Prior to seaming operations, D&E performed trial seams at the beginning of each shift, or upon re-starting the machine after lunch breaks, to demonstrate the adequacy of the seaming apparatus and the operator's procedures. Each trial seam was sampled and tested by D&E for peel adhesion and bonded seam strength. These trial seaming procedures were observed and documented by Golder personnel. Upon observation of successful trial welds, the operators were given approval to begin seaming. Copies of the trial seam logs are presented in Appendix K.3.

In general, the dual-track, hot-wedge fusion method was used for seaming of the HDPE geomembrane liner and ran concurrently with deployment of the geomembrane. This method of fusion seaming produces an air channel that is air-pressure tested for leaks. The extrusion seaming method was utilized for patches, small repairs, and the tie-into the DM-4 liner system. Golder observed and documented the welding of all seams, patches, or other repairs either during or shortly after completion. Copies of the seaming logs are presented in Appendix K.4.

All non-destructive seam continuity testing was performed by D&E and observed by Golder. Non-destructive seam testing was required on all field seams and on all repairs including the destructive test sample patches. Two methods of non-destructive testing were used for this project:

- Vacuum box testing on extrusion welds; and
- Air pressure testing on dual-track, hot-wedge fusion welds.

A vacuum box is a rigid-wall box with a clear Plexiglas top and a neoprene gasket around the bottom of the box forming a seal between the box and the HDPE liner. Vacuum box testing procedures consist of the following:

- Applying a soapy water solution to the seam;
- Applying a vacuum of approximately 10 inches of mercury (5 psi) to the inside of the box for 10 seconds; and
- Observing the weld for bubbles, which would indicate a discontinuity in the weld.

Air pressure testing procedures consist of the following:

- Sealing off the air channel between the inside and outside tracks of the fusion weld at each end of the seam;
- Inserting a needle with an attached pressure gauge into the air channel;
- Inflating the air channel to approximately 40 psi using a small pressurized air tank; and
- Observing the pressure gauge over a five-minute period (a pressure drop of more than 2 psi during this period would indicate a possible discontinuity in the seam); and
- Puncturing of the seam air channel at the far end of the seam to allow release of the pressurized air to verify testing was for the entire seam length.

Any leaks or discontinuities detected in the seams or welds were marked and subsequently repaired in accordance with the specifications. As repairs were made to the geomembrane, Golder documented the location and verified that all repairs were vacuum box tested. Documentation summarizing the observation of the non-destructive seam testing is presented in Appendix K.5.

Repairs consisted of small patches, extrusion beads, or welds. Repairs were made along the intersection of panels, at cuts in the liner made for air pressure testing of the fusion welded seams, or for defects due to holes or blemishes observed in the liner from installation damage. The repairs were marked in the field by Golder and were then subsequently repaired by D&E. A summary of the Repair Logs is presented in Appendix K.6.

A summary of the destructive test results is presented in Appendix K.7. In the destructive test, ten (10) one-inch wide test coupons are cut from each destructive test sample. Five of the coupons are tested for adhesion (peel test mode, both inside and outside track for fusion seams) and five coupons are tested for bonded seam strength (shear test mode) in accordance with ASTM D6392. Breaks are analyzed for Film-Tear-Bond (FTB) or non-FTB in accordance with ASTM D6392.

Destructive test samples were obtained from the HDPE geomembrane seams at a maximum frequency of one sample per 500 lineal feet. A total of 72 destructive test samples were tested, resulting in an overall testing frequency of approximately one test per 491 feet of seam. The majority of theses samples were shipped off-site to Golder's geosynthetics laboratory in Atlanta, Georgia for testing; however, a few were tested in the field or failed without full-scale destructive testing based on preliminary testing performed by the Installer.

Test results indicated that 72 out of 78 destructive seam tests met the project specifications. The seams associated with the six failing destructive tests were either reconstructed or repaired by extrusion welding a cap over the seam.

5.3 Geocomposite

A geocomposite drainage layer was installed directly over the secondary 60-mil HDPE geomembrane liner as the leak detection layer on the floor of the landfill. In addition, a geocomposite drainage layer was installed over the primary 60-mil HDPE geomembrane layer on the side-slope as part of the LCRS.

The geonet component of the geocomposite layers was installed with a minimum 4-inch overlap between adjacent panel edges and fastened using plastic ties at a maximum spacing of 5 feet on panel edges and 1-foot across butt-seams. No butt-seams were placed on the eastern slope. The upper geotextile component was sewn continuously along seams in accordance with the project specifications.

Based on observations made by Golder, the geocomposite layers were installed in accordance with the project specifications.

5.4 Geotextile

An 8-oz/sy non-woven geotextile was installed as a filter layer above the LCRS gravel. D&E installed the geotextile above the LCRS gravel immediately following the placement and grading of the LCRS gravel layer.

The geotextile panels were seamed together with sewing equipment using polymeric thread. Golder verified that adequate seaming was performed and observed the general condition of the geotextile.

5.5 Geosynthetic Clay Liner

A geosynthetic clay liner (GCL) was installed along with a 40-mil HDPE geomembrane backing on the east perimeter levee slope. The geomembrane was installed first as a capillary barrier, and the GCL was then deployed directly on the geomembrane. The geomembrane and GCL were deployed using an all-terrain forklift and a spreader bar. Prior to deployment, the subgrade was inspected by Golder and D&E to verify compliance with the project specifications.

Installation of the 40-mil geomembrane and overlying GCL was started on June 10, 2008 and completed on June 11, 2008. The primary 60-mil HDPE geomembrane was placed as soon as reasonably possible following the GCL installation to protect the GCL from hydration.

6.0 OPERATIONS SOILS

The operations layer soils were placed upon completion of the LCRS gravel, geotextile filter, and LCRS gecomposite drainage layer installation. BPI placed the operations layer soils with transfer dump trucks and a Caterpillar D6 LGP dozer. A grader was used to finish-grade the final operations soil layer surface. The operations soil layer consisted of native borrow soils and a mixture of biosolids and soil, which were placed in specific areas delineated on the construction drawings.

Golder monitored the operations layer soil materials and the soil thickness by observing placement operations and thickness throughout the placement activities. Particle-size distribution tests and moisture content test results completed on the operations soil layer are included in Appendix L.

The placement of the operations layer was started on July 21, 2008. The operations layer and landfill composite liner system was substantially completed on July 31, 2008 with the exception of the outer edges of the liner system. The outer edges of the HDPE geomembrane were left exposed during the completion of the electrical leak location survey as discussed in Section 7. Following the completion of the electrical leak location survey, the contractor finished placing the remaining operations soil layer materials on August 7, 2008.

The as-built plan prepared by Bellecci & Associates, Inc. is presented in Appendix C. Review of the as-built information indicates that the operations layer was constructed in general accordance with the design grades.

7.0 LEAK LOCATION SURVEY

An electrical leak location survey (ELLS) was performed in accordance with ASTM D7007 at the completion of the operations soil layer placement. The ELLS was performed by Leak Location Services, Inc. (LLSI) under subcontract to Golder to determine if any holes or defects existed in the primary 60-mil HDPE geomembrane liner following completion of the LCRS gravel and operations layer. LLSI performed the ELLS from July 30, 2008 through August 1, 2008.

At the beginning of the survey, an artificial leak test was completed by placing a 1/4-inch diameter electrode at the top of the primary geomembrane to verify that the overlying gravels and operations soil could adequately conduct an electrical current. The results of the artificial leak test indicated that overlying materials were adequately conducting an electrical current.

The results of the survey did not detect any defects in the primary HDPE geomembrane. Appendix M includes a report from LLSI that describes the methodology and results of the survey.

8.0 LEACHATE EXTRACTION AND GAS COLLECTION SYSTEMS

Following the construction of the base liner system, the leachate extraction system and the gas collection systems is to be installed. At the time that this report is written, this work is ongoing. The leachate extraction system is to be installed by Advance Wind, Solar, Hydro Power, Inc. (Advanced Power), Redwood Valley, California. The landfill gas (LFG) collection system is to be installed by LFG Control, Inc. (LFGC), from Antioch, California.

The leachate extraction system consists of two Grundfos submersible pumps, installed in the LCRS riser pipe and in the LDS riser pipe. These pumps are automatically controlled by a sump level detection system. Custom flange adaptors were installed on the tops of the riser pipes to provide connection ports for hoses, sampling, and level detection equipment. A hose is provided between the LDS pump and the LCRS riser pipe to remove liquids from the LDS sump. A pipe is extended from the LCRS pump to a leachate storage tank located on the west side of WP-9. This pipe extends above-ground around the northeast perimeter of the landfill. The pumps are powered from a bank of batteries housed in a small shed located adjacent to the riser pipes. The batteries are charged from a bank of solar panels located on the shed roof. As an alternate power source, a generator is also housed in the shed.

The gas collection system consists of a port from the LCRS riser pipe with a valve attachment. A pipe extends from this port to the LFG header pipe that runs along the perimeter levee. At the start of the project, this LFG header was dismantled to allow construction of the perimeter levee for DM-3.1. Following construction, the LFG header is to be reconstructed. The NWSHRL LFG control system was partially constructed during the fall of 2007. It is Golder's understanding that the LFG control system has not yet been put into operation and that no portion of the LFG collection network has been used to-date. Because the LFG header has not been used, no method for gas condensate control within the piping was needed.

9.0 LEAK DETECTION MONITORING CONSIDERATIONS

Water will enter the leak detection system as the liner system is loaded with refuse and the primary low-permeability soil layer consolidates. This is a common occurrence in double-liner systems containing compacted clay liners. This consolidation water is not indicative of a leak in the primary liner system.

The consolidation will generally increase as the refuse loading increases and will significantly decrease after the refuse loading remains constant. Therefore, the occurrence of consolidation water should correlate to increasing refuse loading in the waste cell.

10.0 SUMMARY AND CONCLUSIONS

Golder provided CQA and testing services during construction of the Disposal Module 3.1 base liner system at the NWSHRL in Vacaville, California. Construction of the base liner system occurred between May 8, 2008 and August 7, 2003. Additional construction to install the leachate extraction and gas collection systems began on August 25, 2008 and is currently ongoing. A letter summarizing the installation of these systems will be provided at a later date as an addendum to this report.

The CQA services provided for this project consisted of observing, testing, and documenting the construction activities to verify compliance with the project design plans and specifications. The CQA activities described in this report include the following:

1. Observation and testing the general earthfill soils beneath the liner systems;
2. Observation of the liner subgrade;
3. Observation of the lysimeter and leak detection systems;
4. Observation and testing of the low-permeability soil liner;
5. Observation and testing of the geomembrane, geocomposite, geotextile, and GCL materials;
6. Observation and testing of the LCRS gravel and operations soils construction;
7. Completion of an ELLS; and
8. Review and verification of the containment system as-built documents.

Based on the daily communications with CQA technicians, on observations made during site visits, and on review of the laboratory and field test results and documentation provided and certified by others, Golder hereby states that, in our professional opinion, the containment system for the DM-3.1 base liner at the NWSHRL was constructed in accordance with the project plans and specifications, WDR Order No. R5-2003-0118, and the applicable requirements of the California Code of Regulations, Title 27 pertaining to a Class II Landfill.

Respectfully submitted,

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